

## PART 4

# HYDROLOGIC ENGINEERING STUDIES FOR RESERVOIRS

## Chapter 14 Spillways and Outlet Works

### 14-1. Function of Spillways and Outlet Works

Spillways and outlet works are necessary to provide capability to release an adequate rate of water from the reservoir to satisfy dam safety and water control regulation of the project. Sections 4-2 and 4-3 in EM 1110-2-3600 provide general descriptions of types and operation requirements for spillways and outlet works, respectively.

*a. Spillway adequacy.* While the outflow capability must be provided throughout the operational range of the reservoir, the focus of hydrologic studies is usually on the high flows and spillway adequacy. Dam failures have been caused by improperly designed spillways or by insufficient spillway capacity. Ample capacity is of great importance for earthfill and rockfill dams, which are likely to be destroyed if overtopped; whereas concrete dams may be able to withstand moderate overtopping.

*b. Spillway classification.* Spillways are ordinarily classified according to their most prominent feature, either as it pertains to their shape, location, or discharge channel. Spillways are often referred to as controlled or uncontrolled, depending on whether they are gated or ungated. EM 1110-2-1603 describes a variety of spillway types and provides hydraulic principles, design criteria, and results from laboratory and prototype tests.

*c. Outlet works.* Outlet works serve to regulate or release water impounded by a dam. It may release incoming flows at a reduced rate, as in the case of a detention dam; divert inflows into canals or pipelines, as in the case of a diversion dam; or release stored-water at such rates as may be dictated by downstream needs, evacuation considerations, or a combination of multiple-purpose requirements.

*d. Outlet structure classification.* Outlet structures can be classified according to their purpose, their physical and structural arrangement, or their hydraulic operation. EM 1110-2-1602 provides information on basic hydraulics, conduits for concrete dams, and conduits for earth dams with emphases on flood-control projects. Appendix IV of EM 1110-2-1602 provides an illustrative example of the computation of a discharge rating for outlet works.

*e. Low level outlets.* Low level outlets are provided to maintain downstream flows for all levels of the reservoir

operational pool. The outlets may also serve to empty the reservoir to permit inspection, to make needed repairs, or to maintain the upstream face of the dam or other structures normally inundated.

*f. Outlets as flood-control regulators.* Outlet works may act as a flood-control regulator to release waters temporarily stored in flood control storage space or to evacuate storage in anticipation of flood inflows. In this case, the outflow capacity should be able to release channel capacity, or higher. The flood control storage must be evacuated as rapidly as safely possible, in order to maintain flood reduction capability.

### 14-2. Spillway Design Flood

*a. Spillway design flood analyses.* Spillway design flood (SDF) analyses are performed to evaluate the adequacy of an existing spillway or to size a spillway. For a major project, the conservative practice in the United States is to base the spillway design flood on the probable maximum precipitation (PMP). The PMP is based on the maximum conceivable combination of unfavorable meteorological events. While a frequency is not normally assigned, a committee of ASCE has suggested that the PMP is perhaps equivalent to a return period of 10,000 years.

*b. Probable maximum flood.* The PMF inflow hydrograph is developed by centering the PMP over the watershed to produce a maximum flood response. The unit hydrograph approach, described in Chapter 7 of this manual, is usually applied. Section 13-5 of EM 1110-2-1417 contains information on PMP determination and computation of the PMF.

*c. Flood hydrographs.* The inflow design flood hydrographs are usually for rainfall floods. Normally, such floods will have the highest peak flows but not always the largest volumes. When spillways of small capacities in relation to these inflow design flood peaks are considered, precautions must be taken to ensure that the spillway capacity will be sufficient to evacuate storage so that the dam will not be overtopped by a recurrent storm, and prevent the flood storage from being kept partially full by a prolonged runoff whose peak, although less than the inflow design flood, exceeds the spillway capacity. To meet these requirements, the minimum spillway capacity should be in accord with the following general criteria (Hoffman 1977):

(1) In the case of snow-fed perennial streams, the spillway capacity should never be less than the peak discharge of record that has resulted from snowmelt runoff.

(2) The spillway capacity should provide for the evacuation of sufficient surcharge storage space so that in routing a succeeding flood, the maximum water surface does not exceed that obtained by routing the inflow design flood. In general, the recurrent storm is assumed to begin 4 days after the time of peak outflow obtained in routing the inflow design flood.

(3) In regions having an annual rainfall of 40 in. or more, the time interval to the beginning of the recurrent storm in criterion (2) should be reduced to 2 days.

(4) In regions having an annual rainfall of 20 in. or less, the time interval to the beginning of the recurrent storm in criterion (2) may be increased to 7 days.

### 14-3. Area and Capacity of the Reservoir

*a. Reservoir capacity and operations.* Dam designs and reservoir operating criteria are related to the reservoir capacity and anticipated reservoir operations. The reservoir capacity and reservoir operations are used to properly size the spillway and outlet works. The reservoir capacity is a major factor in flood routings and may determine the size and crest elevation of the spillway. The reservoir operation and reservoir capacity allocations will determine the location and size of outlet works for the controlled release of water for downstream requirements and flood control.

*b. Area-capacity tables.* Reservoir area-capacity tables should be prepared before the final designs and specifications are completed. These area-capacity tables should be based on the best available topographic data and should be the final design for administrative purposes until superseded by a reservoir resurvey. To ensure uniform reporting of data for design and construction, standard designations of water surface elevations and reservoir capacity allocations should be used.

### 14-4. Routing the Spillway Design Flood

*a. Discharge facilities.* The facilities available for discharging inflow from the spillway design flood depend on the type and design of the dam and its proposed use. A single dam installation may have two or more of the following discharge facilities: uncontrolled overflow spillway, gated overflow spillway, regulating outlet, and power plant. With a reservoir full to the spillway crest at the beginning of the design flood, uncontrolled discharge will begin at once. Surcharge storage is created when the outflow capacity is less than the inflow and the excess water goes into storage, causing the pool level to rise above

spillway crest. The peak outflow will occur at maximum pool elevation, which should always be less, to some degree, than the peak inflow.

*b. Gated spillway.* With a gated spillway, the normal operating level is usually near the top of the gates, although at times it may be drawn below this level by other outlets. A gated spillway's main purpose is to maximize available storage and head, while at the same time limiting backwater damages by providing a high initial discharge capacity. In routing the spillway design flood, an initial reservoir elevation at the normal full pool operating level is assumed. Operating rules for spillway gates must be based on careful study to avoid releasing discharges that would be greater than would occur under natural conditions before construction of the reservoir. By gate operation, releases can be reduced and additional water will be held in storage, which is called "induced-surcharge storage." The release rates should be made in accordance with spillway gate regulation schedules developed for each gated reservoir. EM 1110-2-3600 Section 4-5 describes induced surcharge storage and the development and testing of the regulation schedules.

*c. Surcharge storage.* The important factor in the routing procedure is the evaluation of the effect of storage in the upper levels of the reservoir, surcharge storage, on the required outflow capacity. In computing the available storage, the water surface is generally considered to be level. There will be a sloping water surface at the head of the reservoir due to backwater effect, and this condition will create an additional "wedge storage." However, in most large and deep reservoirs this incremental storage can be neglected.

*d. Drawdown.* If a reservoir is drawn down at the time of occurrence of the spillway design flood, the initial increments of inflow will be stored with the corresponding reduction in ultimate peak outflow. Therefore, for maximum safety in design it is generally assumed that a reservoir will be full to the top of flood-control pool at the beginning of the spillway design flood.

*e. Large flood-control storage reservations.* There may be exceptions to the above criteria in the case of reservoirs with large reservations for flood-control storage. However, even in such cases, a substantial part (> 50 percent) of the flood-control storage should be considered as filled by runoff from antecedent floods. The effect on the economics and safety of the project should be analyzed before adopting such assumptions. ER 1110-8-2(FR) contains guidance on inflow design flood development and application.

*f. Release rates.* Assuming a reservoir can be significantly drawn down in advance of the spillway design flood by using a short-term flood warning system is generally not acceptable for several reasons. The volume that can be released is the product of the total rate of discharge at the dam times the warning time. Because the warning time is usually short, except on large rivers, the release rate must be the greatest possible without flood damage downstream. Even under the most favorable conditions, it is unlikely that the released volume will be significant, relative to the volume of the spillway design flood.

#### 14-5. Sizing the Spillway

*a. Storage and spillway capacity.* In determining the best combination of storage and spillway capacity to accommodate the selected inflow design flood, all pertinent factors of hydrology, hydraulics, design, cost, and damage should be considered. In this connection and when applicable, consideration should be given to the following factors:

- (1) The characteristics of the flood hydrograph.
- (2) The damage which would result if such a flood occurred without the dam.
- (3) The damage which would result if such a flood occurred with the dam in-place.
- (4) The damage which would occur if the dam or spillway were breached.
- (5) Effects of various dam and spillway combinations on the probable increase or decrease of damages above or below the dam.
- (6) Relative costs of increasing the capacity of the spillways.
- (7) The use of combined outlet facilities to serve more than one function.

*b. Outflow characteristics.* The outflow characteristics of a spillway depend on the particular device selected to control the discharge. These control facilities may take the form of an overflow weir, an orifice, a tube, or a pipe. Such devices can be unregulated, or they can be equipped with gates or valves to regulate the outflow.

*c. Flood routing.* After a spillway control of certain dimensions has been selected, the maximum spillway discharge and the maximum reservoir water level can be determined by flood routing. Other components of the

spillway can then be proportioned to conform to the required capacity and the specific site conditions, and a complete layout of the spillway can be established. Cost estimates of the spillway and dam can then be made. Estimates of various combinations of spillway capacity and dam height for an assumed spillway type, and of alternative types of spillways, will provide a basis for the selection of the most economical spillway type and the optimum relation of spillway capacity to the height of the dam.

*d. Maximum reservoir level.* The maximum reservoir level can be determined by routing the spillway design flood hydrograph using sequential routing procedures and the proposed operation procedures. This is a basic step in the selection of the elevation of the crest of the dam, the size of the spillway, or both.

*e. Peak rate of inflow.* Where no flood storage is provided, the spillway must be sufficiently large to pass the peak of the flood. The peak rate of inflow is then of primary interest, and the total volume in the flood becomes less important. However, where a relatively large storage capacity above normal reservoir level can be made economically available by a higher dam, a portion of the flood volume can be retained temporarily in reservoir surcharge space, and the spillway capacity can be reduced considerably. If a dam could be made sufficiently high to provide storage space to impound the entire volume of the flood above normal storage level, theoretically, no spillway other than an emergency type would be required, provided the outlet capacity could evacuate the surcharge storage in a reasonable period of time in anticipation of a recurring flood. The maximum reservoir level would then depend entirely on the volume of the flood, and the rate of inflow would be of no concern. From a practical standpoint, however, relatively few sites will permit complete storage of the inflow design flood by surcharge storage.

*f. Overall cost.* The spillway length and corresponding capacity may have an important effect on the overall cost of a project because the selection of the spillway characteristics is based on an economic analysis. In many reservoir projects, economic considerations will necessitate a design utilizing surcharge. The most economical combination of surcharge storage and spillway capacity requires flood routing studies and economic studies of the costs of spillway-dam combinations. Among the many economic factors that may be considered are damage due to backwater in the reservoir, cost-height relations for gates, and utilization in the dam of material excavated from the spillway channel. However, consideration must still be given to the minimum size spillway which must be provided for safety.

*g. Comprehensive study.* The study may require many flood routings, spillway layouts, and spillway and dam estimates. Even then, the study is not necessarily complete because many other spillway arrangements could be considered. A comprehensive study to determine alternative optimum combinations and minimum costs may not be warranted for the design of some dams. Judgment on the part of the designer would be required to select for study only those combinations which show definite advantages, either in cost or adaptability. For example, although a gated spillway might be slightly cheaper than an ungated spillway, it may be desirable to adopt the latter because of its less complicated construction, its automatic and trouble-free operation, its ability to function without an attendant, and its less costly maintenance.

#### 14-6. Outlet Works

*a. Definition.* An outlet works consists of the equipment and structures which together release the required water for a given purpose or combination of purposes. Flows through river outlets and canal or pipeline outlets change throughout the year and may involve a wide range of discharges under varying heads. The accuracy and ease of control are major considerations and a great amount of planning may be justified in determining the type of control devices that can be best utilized.

*b. Description.* Usually, the outlet works consist of an intake structure, a conduit or series of conduits through the dam, discharge flow control devices, and an energy dissipating device where required downstream of the dam. The intake structure includes a trash-rack, an entrance transition, and stop-logs or an emergency gate. The control device can be placed at the intake on the upstream face, at some point along the conduit and be regulated from galleries inside the dam, or at the downstream end of the conduit with the operating controls placed in a gate-house on the downstream face of the dam. When there is a power plant or other structure near the face of the dam, the outlet conduits can be extended farther downstream to discharge into the river channel beyond these features. In this case, a control valve may be placed in a gate structure at the end of the conduit.

*c. Discharge.* Discharges from a reservoir outlet works fluctuate throughout the year depending upon downstream water needs and reservoir flood control requirements. Therefore, impounded water must be released at specific regulated rates. Operating gates and regulating valves are used to control and regulate the outlet works flow and are designed to operate in any position from closed to fully open. Guard or emergency gates are

designed to close if the operating gates fail, or where dewatering is desired to inspect or repair the operating gates.

*d. Continuous low-flow releases.* Continuous low-flow releases are usually required to satisfy the needs of fish, wildlife and existing water rights downstream from the dam. When the low-flow release is small, one or two separate small bypass pipes, with high-pressure regulating valves, are provided to facilitate operations. Flood-regulating gates may be used for making low-flow releases when those low-flow releases require substantial gate openings (EM 1110-2-1602).

*e. Uses of an outlet works.* An outlet works may be used for diverting the river flow or portion thereof during a phase of the construction period, thus avoiding the necessity for supplementary installations for that purpose. The outlet structure size dictated by this use rather than the size indicated for ordinary outlet requirements may determine the final outlet works capacity.

*f. Intake level.* The establishment of the intake level is influenced by several considerations such as maintaining the required discharge at the minimum reservoir operating elevation, establishing a silt retention space, and allowing selective withdrawal to achieve suitable water temperature and/or quality. Dams which will impound waters for irrigation, domestic use, or other conservation purposes must have the outlet works intake low enough to be able to draw the water down to the bottom of the allocated storage space. Further, if the outlets are to be used to evacuate the reservoir for inspection or repair of the dam, they should be placed as low as practicable. However, it is usual practice to make an allowance in a reservoir for inactive storage for silt deposition, fish and wildlife conservation, and recreation.

*g. Elevation of outlet intake.* Reservoirs become thermally stratified, and taste and odor vary between elevations. Therefore, the outlet intake should be established at the best elevation to achieve satisfactory water quality for the purpose intended. Downstream fish and wildlife requirements may determine the temperature at which the outlet releases should be made. Municipal and industrial water use increases the emphasis on water quality and requires the water to be drawn from the reservoir at the elevation which produces the most satisfactory combination of odor, taste, and temperature. Water supply releases can be made through separate outlet works at different elevations if requirements for the individual water uses are not the same and the reservoir is stratified.

*h. Energy-dissipating devices.* The two types of energy dissipating devices most commonly used in conjunction with outlet works on concrete dams are hydraulic jump stilling basins and plunge pools. On some dams, it is possible to arrange the outlet works in conjunction with the

spillway to utilize the spillway-stilling device for dissipating the energy of the water discharging from the river outlets. Energy-dissipating devices for free-flow conduit outlet works are essentially the same as those for spillways.

## Chapter 15

### Dam Freeboard Requirements

#### 15-1. Basic Considerations

*a. Freeboard.* Freeboard protects dams and embankments from overflow caused by wind-induced tides and waves. It is defined as the vertical distance between the crest of a dam and some specified pool level, usually the normal operating level or the maximum flood level. Depending on the importance of the structure, the amount of freeboard will vary in order to maintain structural integrity and the estimated cost of repairing damages resulting from overtopping. Riprap or other types of slope protection are provided within the freeboard to control erosion that may occur even without overtopping.

*b. Estimating freeboard.* Freeboard is generally based on maximum probable wind conditions when the reference elevation is the normal operating level. When estimating the freeboard to be used with the probable maximum reservoir level, a lesser wind condition is used because it is improbable that maximum wind conditions will occur simultaneously with the maximum flood level. A first step in wave height determinations is a study of available wind records to determine velocities and related durations and directions. Three basic considerations are generally used in establishing freeboard allowance. These are wave characteristics, wind setup, and wave runoff.

*c. Further information.* The Corps of Engineers Coastal Engineering Research Center (CERC) has developed criteria and procedures for evaluating each of the above areas. The primary references are EM 1110-2-1412 and EM 1110-2-1414. The procedures presented in these manuals have received general acceptance for use in estimating freeboard requirements for reservoirs.

*d. Applications.* In applications for inland reservoirs, it is necessary to give special consideration to the influences that reservoir surface configuration, surrounding topography, and ground roughness may have on wind velocities and directions over the water surface. The effects of shoreline irregularities on wave refraction and influences of water depth variations on wave heights and lengths must be accounted for. Although allowances can only be approximated, the estimates of wave and wind tide characteristics in inland reservoirs can be prepared sufficiently accurate for engineering purposes.

#### 15-2. Wind Characteristics over Reservoirs

*a. General.* The more violent windstorms experienced in the United States are associated with tropical storms (hurricanes) and tornadoes. Hurricane wind characteristics may affect reservoir projects located near Atlantic and Gulf coastlines, but winds associated with tornadoes are not applicable to the determination of freeboard allowances for wave action. In mountainous regions, the flow of air is influenced by topography as well as meteorological factors. These "orographic" wind effects, when augmented by critical meteorological patterns, may produce high wind velocities for relatively long periods of time. Therefore, they should be given special consideration in estimating wave action in reservoirs located in mountainous regions. In areas not affected by major topographic influences, air movement is generally the result of horizontal differences in pressure which in turn are due primarily to large-scale temperature differences in air masses. Wind velocities and durations associated with these meteorological conditions, with or without major influences of local topography, are of major importance in estimating wave characteristics in reservoirs.

*b. Isovel patterns.* Estimates of wind velocities and directions near a water surface at successive intervals of time, as a windstorm passes the area, may be established by deriving "isovel" patterns. Sequence relations can represent wind velocities at, say, one-half hour intervals during periods of maximum winds, and one-hour or longer intervals thereafter. The "isovel" lines connect points of equal wind speeds, resembling elevation contour maps. Wind directions are indicated by arrows. EM 1110-2-1412, Sections 1.9 and 1.10 describe storms and the storm surge generation process. Figure 1-1a shows an example wind isovel pattern and pertinent parameters.

*c. Relation of wind duration to wave heights.* If wind velocity over a particular fetch remains constant, wave heights will progressively increase until a limiting maximum value is attained, corresponding approximately to relations dependent on fetch distance, wind velocity, and duration. Accordingly, wind velocity-duration relations applicable to effective reservoir fetch areas are needed for use in computing wave characteristics in reservoirs.

*d. Wind velocity-duration relations.* In some cases it is desired to estimate wave characteristics in existing reservoirs in order to analyze causes of riprap damage or for other reasons. Wind records, supplemented by meteorological studies are usually required. Data on actual

windstorms of record have been maintained at many U.S. Weather Bureau stations. Index values, such as the fastest mile, 1-min average or 5-min average velocities, with direction indications, are usually presented in climatological data publications. Some data collected by other agencies and private observers may be available in published or unpublished form. However, information regarding wind velocities sustained for several hours or days is not ordinarily published in detail. Accordingly, special studies are usually required to determine wind velocity-duration relations applicable to specific effective fetch areas involved in wave computations. Basic records for such studies are usually available from the U.S. Weather Bureau offices or other observer stations. Some summaries of wind velocities over relatively long periods of time have been published by various investigators, and others may be available in project reports related to water resources development.

*e. Generalized wind velocity-duration relations.* Studies show that maximum wind velocities in one general direction during major windstorms, in most regions of the United States, have averaged approximately 40 to 50 mph for a period of 1 hr. Corresponding velocities in the same general direction for periods of 2 hr and 6 hr have averaged 95 percent and 88 percent, respectively, of the maximum 1-hr average velocity. In EM 1110-2-1414, Figure 5-26 provides the ratio of wind speed of any duration to the 1-hr wind speed. Extreme wind velocities for brief periods, normally referred to as "fastest mile" or 1-min average, have been recorded as high as 150 to 200 percent of maximum 1-hr averages in most regions. In EM 1110-2-1414, Figures 5-18 through 5-20 provide the annual extreme fastest-mile speed 30 ft (9.1 m) above ground for the 25-year, 50-year, and 100-year recurrence intervals, respectively. However, these extreme values are seldom of interest in computing wave characteristics in reservoirs. Generalized wind velocity-duration relations are considered to be fairly representative of maximum values that are likely to prevail over a reservoir in generally a single direction for periods up to 6 hr (excluding projects located in regions that are subject to severe hurricanes or orographic wind-flow effects). Special studies of wind characteristics associated with individual project areas should be made when determinations of unusual importance, or problems requiring consideration of wind durations exceeding 6 hr, are involved.

*f. Ratio of wind velocities over water and land areas.* The wind velocities described in paragraph *d* are for over land. Under comparable meteorological conditions, wind velocities over water are higher than over land surfaces because of smoother and more uniform surface conditions. Winds blowing from land tend to increase with passage

over reservoir areas, and vice versa. The relationships are not constant, but vary with topographic and vegetative cover of land areas involved, reservoir configurations, and other conditions affecting air flow. However, on the basis of research and field studies (Technical Memorandum No. 132, USACE 1962), the following ratios represent averages that are usually suitable for computing wave characteristics in reservoirs that are surrounded by terrain of moderate irregularities and surface roughness:

Fetch ( $F_e$ ) in Miles	Wind ratio <u>Over Water</u> Over Land
0.5	1.08
1	1.13
2	1.21
3	1.26
4	1.28
5 (or over)	1.30

### 15-3. Computation of Wave and Wind Tide Characteristics

*a. Effective wind fetch ( $F_e$ ) for wave generation.* The characteristics of wind-generated waves are influenced by the distance that wind moves over the water surface in the "fetch" direction. The generally narrow irregular shoreline of inland reservoirs will have lower waves than an open coast because there is less water surface for the wind to act on. The method to compensate for the reduced water surface for an enclosed body of water is computation of an effective fetch. The effective fetch ( $F_e$ ) adjusts radial lines from the embankment to various points on the reservoir shore. The radials spanning 45 deg on each side of the central radial are adjusted by the cosign of their angle to the central radial to estimate an average effective fetch. The computation procedure is shown in EM 1110-2-1414, Figure 5-33 and Example Problem 7-2. Generalized relations are based on effective fetch distances derived in this manner.

*b. Fetch distance for wind tide computations.* Fetch distances for use in estimating wind tide (set-up) effects are usually longer than effective fetch distances used in estimating wave heights. In as much as wind tide effects in deep inland reservoirs are relatively small, extensive studies to refine estimates are seldom justified. For practical purposes, it is usually satisfactory to assume that the wind tide fetch is equal to twice the effective fetch ( $F_e$ ). If wind tide heights determined in this manner are relatively large in relation to overall freeboard requirements, more detailed analyses are advisable using methods as generally discussed in Chapter 3 of EM 1110-2-1414.

c. *Generalized diagrams for wave height and wave period in deep inland reservoirs.* EM 1110-2-1414, Figure 5-34, presents generalized relations between significant wave height, wave period, fetch ( $F_o$ ), and wind velocities corresponding to critical durations. These diagrams were developed from research and field studies based on wind speed at 10 m (33 ft). If wind speeds are for a different level, Equation 5-12 can be used to adjust to the 10 m level (EM 1110-2-1414).

d. *Wave characteristics in shallow inland lakes and reservoirs.* In the analyses of wave characteristics, lakes and reservoirs are considered to be shallow when depths in the wave-generating area are generally less than about one-half the theoretical deep-water wave length ( $L_o$ ) corresponding to the same wave period ( $T$ ). Curves presented in Figures 5-35 through 5-44 represent relations between wave characteristics, fetch distances (in feet) and constant water depths in the wave-generating area, ranging from 5 to 50 ft (1.5 to 15.2 m) (EM 1110-2-1414).

e. *Wind tides (set-up) in inland waters.*

(1) When wind blows over a water surface, it exerts a horizontal stress on the water, driving it in the direction of the wind. In an enclosed body of water, this wind effect results in a piling up of water at the leeward end, and a lowering of water level at the windward end. This effect is called "wind tide" or "wind set-up." Wind set-up can be reasonably estimated for lakes and reservoirs, based on the following equation:

$$S = \frac{U^2 F}{1400 D} \quad (15-1)$$

in which  $S$  is wind tide (set-up) in feet above the stillwater level that would prevail with zero wind action;  $U$  is the average wind velocity in statute miles per hour over the fetch distance ( $F$ ) that influences wind tide;  $D$  is the average depth of water generally along the fetch line (EM 1110-2-1414). The fetch distance ( $F$ ) used in the above formula is usually somewhat longer than the effective fetch ( $F_o$ ) used in wave computations, as indicated in paragraph 15-3b. Refer to EM 1110-2-1414, Section 3-2 for a discussion of prediction models.

#### 15-4. Wave Runup on Sloping Embankment

a. *Introduction.* Most dam embankments are fronted by deep water, have slopes between 1 on 2 and 1 on 4, and are armored with riprap. Rock-fill dams are considered as permeable rubble slopes and earth-fill dams with riprap armor are considered impermeable. Laboratory tests of

many slopes, wave conditions, and embankment porosity provide sufficient data to make estimates of wave runup on a prototype embankment.

b. *Relative runup relations.* EM 1110-2-2904 Plate 25 presents generalized relations on wave runup on rubble-mound breakwaters and smooth impervious slopes. Plate 26 provides similar curves for various embankment slopes for water depths greater than three-times wave height. The curves correspond to statistical averages of a large number of small and large-scale hydraulic model test results, and have been adjusted for model scale effects to represent prototype conditions. The relations were based primarily on tests involving mechanically generated waves and may differ somewhat from relations associated with individual waves in natural wind-generated spectrums of waves. However, general field observations and comparisons with wave experiences support the conclusion that relations presented.

c. *Runup of waves on sloping embankments.*

(1) If waves generated in deep water (i.e., depths exceeding about one-third to one-half the wave length), reach the toe of an embankment without breaking, the vertical height of runup may be computed by multiplying the deep-water wave height ( $H_o$ ) by the relative runup ratio ( $R/H$ ) obtained from EM 1110-2-2904, Plate 26, for the appropriate slope and wave steepness ( $H_o/L_o$ ). In this case, deep-water values of  $H_o$  and  $L_o$  should be used as indices, even though wave heights and lengths are modified by passing through areas in which water depths are less than  $L_o/3$  (provided the depth is not small enough to cause the wave to break before reaching the embankment). That is, the height of runup may be computed by using the deep-water steepness  $H_o/L_o$ , whether the structure under study is located in deep water or in shallow water, provided the wave does not break before reaching the toe of the structure.

(2) If waves are generated primarily by winds over open-water areas where the relative depth ( $d/L$ ) is appreciably less than 0.3, the wave heights and periods should be computed by procedures applicable to shallow waters.

(3) Waves generated by wind over open-water areas of a particular depth change characteristics when they reach areas where the constant depth is substantially less, the height ( $H$ ) tending to increase while the length ( $L$ ) decreases. The distribution of wave energy changes as a wave enters the shallow water, the proportion of total energy which is transmitted forward with the wave toward the shore increasing, although the actual amount of this

translated energy remains constant except from minor frictional effects. If the depth continues to decrease, the steepness ratio ( $H/L$ ) increases, until finally the wave becomes unstable and breaks, resulting in appreciable energy dissipation. Theoretically, the maximum wave cannot exceed  $0.78 D$ , where  $D$  is the depth of water without wave action. After breaking, the waves will tend to reform with lower heights within a distance equal to a few wave lengths. For most engineering applications, it is satisfactory to assume that the wave height after breaking will equal approximately  $0.78 D$  in the shallow area and that  $L$  will be the same as before the wave broke. Plate 26 would then be entered with a wave-steepness ratio equal to  $0.78/L$  to determine the relative runup ratio ( $R/H$ ), and this ratio would be multiplied by  $0.78 D$  to obtain the estimated runup height ( $R$ ). This procedure should provide conservative results under circumstances in which the distance between the point where waves reach breaking depths and location of the structure under study is long enough to permit waves to reform, and short enough to preclude substantial build-up by winds prevailing over the shallower area. More accurate values could be obtained by using this breaking height ( $0.78 D$ ) and period to obtain comparable deep water values of  $H_o$  and  $L_o$ .

*d. Adjustments in wave runup estimates for variations in riprap.*

(1) A rough riprap layer on an embankment tends to reduce the height of runup after a wave breaks. If the riprap layer thickness is small in comparison with wave magnitudes and the underlying surface is relatively impermeable, so that the void spaces in the riprap remain mostly filled with water between successive waves during severe storm events, the height of runup may closely approach heights attained on smooth embankments of comparable slope. However, if the riprap layer is sufficiently rough, thick, and free draining to quickly absorb the water that impinges on the embankment as each successive wave breaks, further wave runup will be almost completely eliminated.

(2) The design of riprap to absorb most of the energy of breaking waves is practicable if waves involved will be relatively small or moderate, but costs and other practical considerations usually preclude such design where large waves are encountered. Accordingly, the design characteristics of riprap layers are usually somewhere between the two extremes described above.

## 15-5. Freeboard Allowances for Wave Action

*a. Purpose.* In connection with the design of dams and reservoirs, the estimate of freeboard is required to

establish allowances needed to provide for wave action that is likely to affect various project elements, as follows:

- (1) Main embankment of the dam, and supplemental dike sections.
- (2) Levees that protect areas within potential flowage limits of the reservoir.
- (3) Highway and railroad embankments that intersect the reservoir limits.
- (4) Structures located within the reservoir area.
- (5) Shoreline areas that are subject to adverse effects of wave action.

*b. Freeboard on dams.* The establishment of freeboard allowances on dams includes not only the consideration of potential wave characteristics in a reservoir, but several other factors of importance, including certain policy matters.

*c. Freeboard allowances for wave action on embankments and structures within reservoir flowage limits.*

(1) Wave action effects must be taken into account in establishing design grades and slope protection measures for highway, railroad, levee, and other embankments that intersect or border a reservoir. The design of operating structure, boat docks, recreational beaches, and shoreline protection measures at critical locations involves the consideration of wave characteristics and frequencies under a range of conditions. Estimates of wave characteristics affecting the design of these facilities can have a major influence on the adequacy of design and costs of relocations required for reservoir projects, and in the development of supplemental facilities.

(2) The freeboard reference level selected as a base for estimating wave effects associated with each of the several types of facilities referred to above will be governed by considerations associated with the particular facility. Otherwise, procedures generally as described with respect to the determination of freeboard allowances for dams should be followed, and stage hydrographs and related wave runup elevations corresponding to the selected wind criteria should be prepared. However, the freeboard reference level and coincident wind velocity-duration relations selected for these studies usually correspond to conditions that would be expected with moderate frequency, instead of the rare combinations assumed in estimating the height of dam required for safety.

(3) In estimating effects of wave action on embankments and structures, the influences of water depths near the facility should be carefully considered. If the shallow depths prevail for substantial distances from the embankment or structure under study, wave effects may be greatly reduced from those prevailing in deep-water areas. On the other hand, facilities located where sudden reductions in water depths cause waves to break are likely to be subjected to greater dynamic forces than would be imposed on similar facilities located in deep water. This

consideration is particularly important in estimating the effects waves may have on bridge structures that are partially submerged under certain reservoir conditions.

(4) Systematic analyses of wave effects associated with various key locations along embankments that cross or border reservoirs provide a practical basis for varying design grades and erosion protection measures to establish the most economical plan to meet pertinent operational and maintenance standards.

## Chapter 16 Dam Break Analysis

### 16-1. Introduction

*a. Corps policy.* It is the policy of the Corps of Engineers to design, construct, and operate dams safely (ER 1110-8-2(FR)). When a dam is breached, catastrophic flash flooding occurs as the impounded water escapes through the gap into the downstream channel. Usually, the response time available for warning is much shorter than that for precipitation-runoff floods, so the potential for loss of life and property damage is much greater.

*b. Hazard evaluation.* A hazard evaluation is the basis for selecting the performance standards to be used in dam design or in evaluating existing dams. When flooding could cause significant hazards to life or major property damage, the design flood selected should have virtually no chance of being exceeded. ER 1110-8-2(FR) provides dam safety standards with respect to the appropriate selection of an inflow design flood. If human life is at risk, the general requirement is to compute the flood using PMP. If lesser hazards are involved, a smaller flood may be selected for design. However, all dams should be designed to withstand a relatively large flood without failure even when there is apparently no downstream hazard involved under present conditions of development.

*c. Safety design.* Safety design includes studies to ascertain areas that would be flooded during the design flood and in the event of dam failure. The areas downstream from the project should be evaluated to determine the need for land acquisition, flood plain management, or other methods to prevent major damage. Information should be developed and documented suitable for releasing to downstream interests regarding the remaining risks of flooding.

*d. National Dam Safety Act.* The potential for catastrophic flooding due to dam failures in the 1960's and 1970's brought about passage of the National Dam Safety Act, Public Law 92-367. The Corps of Engineers became responsible for inspecting U.S. Federal and non-Federal dams, which met the size and storage limitations of the act, in order to evaluate their safety. The Corps inventoried dams; surveyed each State and Federal agency's capabilities, practices, and regulations regarding the design, construction, operation, and maintenance of the dams; developed guidelines for the inspection and evaluation of dam safety; and formulated recommendations for a comprehensive national program.

*e. Flood emergency documents.* In support of the National Dam Safety Program, flood emergency planning for dams was evaluated in the 1980's, and a series of documents were published: *Emergency Planning for Dams, Bibliography and Abstracts of Selected Publications* (HEC 1982a), *Flood Emergency Plan, Guidelines for Corps Dams*, Research Document 13 (HEC 1980), *Example Emergency Plan for Blue Marsh Dam and Lake* (HEC 1983a), and *Example Plan for Evacuation of Reading, Pennsylvania in the Event of Emergencies at Blue Marsh Dam and Lake* (HEC 1983b). The development of an emergency plan requires the identification of the type of emergencies to be considered, the gathering of needed data, performing the analyses and evaluations, and presenting the results. HEC Research Document 13 (HEC 1980) provides guidelines for each step of the process.

### 16-2. Dam Breach Analysis

*a. Causes of dam failures.* Dam failures can be caused by overtopping a dam due to insufficient spillway capacity during large inflows to the reservoir, by seepage or piping through the dam or along internal conduits, slope embankment slides, earthquake damage and liquefaction of earthen dams from earthquakes, or landslide-generated waves within the reservoir. Hydraulics, hydrodynamics, hydrology, sediment transport mechanics, and geotechnical aspects are all involved in breach formation and eventual dam failure. HEC Research Document 13 lists the prominent causes as follows:

- (1) Earthquake.
- (2) Landslide.
- (3) Extreme storm.
- (4) Piping.
- (5) Equipment malfunction.
- (6) Structural damage.
- (7) Foundation failure.
- (8) Sabotage.

*b. Dam breach characteristics.* The breach is the opening formed in the dam when it fails. Despite the fact that the main modes of failure have been identified as piping or overtopping, the actual failure mechanics are not well understood for either earthen or concrete dams. In previous attempts to predict downstream flooding due to dam failures, it was usually assumed that the dam failed

completely and instantaneously. These assumptions of instantaneous and complete breaches were used for reasons of convenience when applying certain mathematical techniques for analyzing dam-break flood waves. The presumptions are somewhat appropriate for concrete arch-type dams, but they are not suitable for earthen dams and concrete gravity-type dams.

(1) Earthen dams, which exceedingly outnumber all other types of dams, do not tend to completely fail, nor do they fail instantaneously. Once a developing breach has been initiated, the discharging water will erode the breach until either the reservoir water is depleted or the breach resists further erosion. The fully formed breach in earthen dams tends to have an average width ( $b$ ) in the range ( $h_d < b < 3h_d$ ) where,  $h_d$  is the height of the dam. Breach widths for earthen dams are therefore usually much less than the total length of the dam as measured across the valley. Also, the breach requires a finite interval of time for its formation through erosion of the dam materials by the escaping water. The total time of failure may range from a few minutes to a few hours, depending on the height of the dam, the type of materials used in construction, and the magnitude and duration of the flow of escaping water. Piping failures occur when initial breach formation takes place at some point below the top of the dam due to erosion of an internal channel through the dam by escaping water. As the erosion proceeds, a larger and larger opening is formed. This is eventually hastened by caving-in of the top portion of the dam.

(2) Concrete gravity dams also tend to have a partial breach as one or more monolith sections formed during the dam construction are forced apart by the escaping water. The time for breach formation is in the range of a few minutes.

(3) Poorly constructed earthen dams and coal-waste slag piles which impound water tend to fail within a few minutes and have average breach widths in the upper range or even greater than those for the earthen dams mentioned above.

*c. Dam breach parameters.* The parameters of failure depend on the dam and the mode of failure. For flood hydrograph estimation, the breach is modeled assuming weir conditions, and the breach size, shape, and timing are the important parameters. The larger the breach opening and the shorter the time to total failure, the larger the peak outflow. HEC Research Document 13, Table 1, lists suggested breach parameters for earth-fill, concrete-gravity, and concrete-arch dams. There are two basic approaches used to determine possible breach sizes and times.

(1) The first approach uses statistically derived regression equations, like those formulated by MacDonald and Langridge-Monopolis (1984) and by Froelich (1987). Both sets of equations are based on actual data from dozens of historic dam failures. The MacDonald, and Langridge-Monopolis study was based on data from 42 constructed earth- and rock-fill dams. The Froelich study included data from constructed and landslide-formed earthen dams. Both studies resulted in a set of graphs and equations that can be used to predict the approximate size of the breach and the time it takes for the breach to reach its full size.

(2) The second approach is a physically based computer model called BREACH, developed by Dr. Danny Fread (1989) for the National Weather Service. The breach model uses sediment transport and hydraulic routing equations to simulate the formation of either a piping or overtopping type of failure. The model requires information about the physical dimensions of the dam, as well as a detailed description of the soil properties of the dam. Soils information includes D50 (mm), porosity, unit weight ( $\text{lb/ft}^3$ ), internal friction angle, cohesive strength ( $\text{lb/ft}^2$ ), and D90/D30. These parameters can be specified separately for the inner-core and outside-bank materials of a dam.

### 16-3. Dam Failure Hydrograph

*a. Flow hydrograph.* The flow hydrograph from a breached dam may be computed using traditional methods for flow routing through a reservoir and downstream channel. The reservoir routing approach is the same as routing for the spillway design flood, described in Chapter 14. Generally, a short time step is required because the breach formation and resulting reservoir outflow change rapidly with time.

*b. Routing methods.* The choice between hydraulic and hydrologic routing depends on many factors, including the nature of available data and accuracy required. The hydraulic method is the more accurate method of routing the unsteady flow from a dam failure flood through the downstream river. This technique simultaneously computes the discharge, water surface elevation, and velocity throughout the river reach. Chapter 9 of EM 1110-2-1417 describes the routing methods and applicability of routing techniques. Chapter 5 of EM 1110-2-1416 describes unsteady flow computations.

*c. Geometry and surface area.* The geometry and surface area of the reservoir can also affect the choice of method. For very narrow and long reservoirs where the dam is relatively large, the change of water level at the

failed dam is rapid, and the unsteady flow method is useful. However, for very large reservoirs where the dam is small compared to the area of the lake, the change in water level is relatively slow and the storage routing method (Modified Puls) is economical in developing the failure hydrograph. Because of the rapid change in water level, small time periods are required for both methods.

*d. Height of downstream water.* The height of the water downstream of a dam (tailwater) also affects the outflow hydrograph in a failure analysis. It also affects the formation or nonformation of a bore in front of the wave.

*e. Deriving the peak outflow.* By assuming a rectangular cross section, zero bottom slope, and an instantaneous failure of a dam, the peak outflow can be derived by the mathematical expression originally developed by St. Venant, as follows:

$$Q_{\max} = \frac{8}{27} W_b \sqrt{g} Y_o^{3/2} \quad (16-1)$$

where

$Y_o$  is the initial depth,  $W_b$  is the width of the breach,  $g$  is the gravity coefficient, and the water depth,  $y$ , just downstream of the dam is

$$y = \frac{4}{9} Y_o \quad (16-2)$$

This equation is applicable only for relatively long and narrow rectangular channels where the dam is completely removed. *Guidelines for Calculating and routing a Dam-Break Flood*, HEC Research Document No. 5 (HEC 1977) describes this approach.

*f. Failed dam outflow hydrograph.* The outflow hydrograph from a failed dam may also be approximated by a triangle. For instantaneous failure, a right triangle is applicable. The base represents the time to empty the reservoir volume, and the height represents the instantaneous peak outflow. In erosion analysis, the Office of Emergency Services, after consultation with other agencies, suggested an isosceles triangle. The rising side of the isosceles triangle is developed by assuming that half of the reservoir storage is required to erode the dam to natural ground level. The apex of the triangle represents the peak flow through the breach under the assumption that the flow occurs at critical depth.

*g. Potential for overtopping.* The Hydrologic Engineering Center's HEC-1 Flood Hydrograph Package (HEC 1990c) can be used to determine the potential for overtopping of dams by run off resulting from various proportions of the PMF. This technique is most appropriate for simulating breaches in earthen dams caused by overtopping. Other conditions may be approximated, however, such as instantaneous failure. This method makes six assumptions:

- (1) Level-pool reservoir routing to determine time-history of pool elevation.
- (2) Breach shape is a generalized trapezoid with bottom width and side slopes prespecified by the analyst.
- (3) Bottom of the breach moves downward at a constant rate.
- (4) Breach formation begins where the water surface in the reservoir reaches a prespecified elevation.
- (5) Breach is fully developed when the bottom reaches a prespecified elevation.
- (6) Discharge through the breach can be calculated independently of downstream hydraulics, i.e., critical depth occurs at or near the breach. A tailwater rating curve or a single cross section (assuming normal-depth for a rating) can be used to simulate submergence effects.

The total discharge from the dam at any instant is calculated by summing the individual flows through the low level outlet, over the spillway and top of the dam, and through the breach.

*h. Peak flow values.* With several calculations of theoretical flood peaks from assumed breaches, peak flow values may seem either too low or too high. One way of checking the reasonableness of the assumption is to compare the calculated values with historical failures. An envelope of estimated flood peaks from actual dam failures prepared by the Bureau of Reclamation is a good means of comparing such values. HEC Research Document No. 13, Figure 2, provides an envelope of experienced outflow rates from breached dams, as a function of hydraulic depth.

#### 16-4. Dam Break Routing

*a. Dam-break flood hydrographs.* Dam-break flood hydrographs are dynamic, unsteady flow events.

Therefore, the preferred routing approach is to utilize a full unsteady flow routing model. The HEC-1 Flood Hydrograph package provides the capability to compute and route the inflow design flood and compute the breach and resulting hydrograph, but its channel routing is limited to hydrologic methods. The most appropriate HEC-1 approach is the Muskingum-Cunge option. The option uses a simple cross section plus reach slope and length to define a routing reach. No downstream backwater effects are considered. If simplified representations of the downstream river reaches are acceptable, an adequate routing may be obtained.

*b. St. Venant equations.* The St. Venant equations apply to gradually varied flow with a continuous profile. If features which control or interrupt the water surface profile exist along the main stem of the river or its tributaries, internal boundary conditions are required. These features include dams, bridges, roadway embankments, etc. If the structure is a dam, the total discharge is the sum of spillway flow, flow over the top of the dam, gated-spillway flow, flow through turbines, and flow through a breach, should a breach occur. The spillway flow and dam overtopping are treated as weir flow, with corrections for submergence. The gated outlet can represent a fixed gate or one in which the gate opening can vary with time. These flows can also be specified by rating curves which define discharge passing through the dam as a function of upstream water surface elevation.

*c. Unsteady flow computer programs.* There are an increasing number of available unsteady flow computer programs. The FLDWAV program is a generalized unsteady-flow simulation model for open channels. It replaces the DAMBRK, DWOPER, and NETWORK models, combining their capabilities and providing new hydraulic simulation procedures within a more user-friendly model structure (DeVries and Hromadka 1993). Given the long history of application by the National Weather Service, this program is likely the most capable for this purpose.

*d. FLDWAV.* FLDWAV can simulate the failure of dams caused by either overtopping or piping failure of the dam. The program can also represent the failure of two or more dams located sequentially on a river. The program is based on the complete equations for unsteady open-channel flow (St. Venant equations). Various types of external and internal boundary conditions are programmed into the model. At the upstream and downstream boundaries of the model (external boundaries), either discharges or water surface elevations, which vary with time, can be specified.

*e. Special features.* The following special features and capacities are included in FLDWAV: variable  $\Delta t$  and  $\Delta x$  computational intervals; irregular cross-sectional geometry; off-channel storage; roughness coefficients that vary with discharge or water surface elevation, and with distance along the waterway; capability to generate linearly interpolated cross sections and roughness coefficients between input cross sections; automatic computation of initial steady flow and water elevations at all cross sections along the waterway; external boundaries of discharge or water surface elevation time series (hydrographs), a single-valued or looped depth-discharge relation (tabular or computed); time-dependent lateral inflows (or outflows); internal boundaries enable treatment of time-dependent dam failures, spillway flows, gate controls, or bridge flows, or bridge-embankment overtopping flow; short-circuiting of floodplain flow in a valley with a meandering river; levee failure and/or overtopping; a special computational technique to provide numerical stability when treating flows that change from supercritical to subcritical, or conversely, with time and distance along the waterway; and an automatic calibration technique for determining the variable roughness coefficient by using observed hydrographs along the waterway.

*f. UNET.* The unsteady flow program UNET (HEC 1995) has a dam-break routing capability. However, there has been limited application of this feature. UNET could be used to route the outflow hydrograph computed in an HEC-1 runoff-dam break model. Both programs can read and write hydrographs using the HEC Data Storage System, HEC-DSS (HEC 1995a).

## 16-5. Inundation Mapping

*a. Preparation of maps.* To evaluate the effects of dam failure, maps should be prepared delineating the area which would be inundated in the event of failure. Land uses and significant development or improvements within the area of inundation should be indicated. The maps should be equivalent to or more detailed than the USGS quadrangle maps, 7.5-min series, or of sufficient scale and detail to identify clearly the area that should be evacuated if there is evident danger of failure of the dam. Copies of the maps should be distributed to local government officials for use in the development of an evacuation plan. The intent of the maps is to develop evacuation procedures in case of collapse of the dam, so the travel time of the flood wave should be indicated on every significant habitation area along the river channel.

*b. Evaluation of hazard potential.* To assist in the evaluation of hazard potential, areas delineated on inundation maps should be classified in accordance with the degree of occupancy and hazard potential. The potential for loss of life is affected by many factors, including but not limited to the capacity and number of exit roads to higher ground and available transportation. Hazard potential is greatest in urban areas. The evaluation of hazard potential should be conservative because the extent of inundation is usually difficult to delineate precisely.

*c. Hazard potential for recreation areas.* The hazard potential for affected recreation areas varies greatly, depending on the type of recreation offered, intensity of use, communications facilities, and available transportation. The potential for loss of life may be increased where recreationists are widely scattered over the area of potential inundation because they would be difficult to locate on short notice.

*d. Industries and utilities.* Many industries and utilities requiring substantial quantities of water are located on or near rivers or streams. Flooding of these areas and industries, in addition to causing the potential for loss of life, can damage machinery, manufactured products, raw materials and materials in process of manufacture, plus interrupt essential community services.

*e. Least hazard potential.* Rural areas usually have the least hazard potential. However, the potential for loss of life exists, and damage to large areas of intensely cultivated agricultural land can cause high economic loss.

*f. Evacuation plans.*

(1) Evacuation plans should be prepared and implemented by the local jurisdiction controlling inundation areas. The assistance of local civil defense personnel, if available, should be requested in preparation of the evacuation plan. State and local law enforcement agencies usually will be responsible for the execution of much of the plan and should be represented in the planning effort. State and local laws and ordinances may require that other state, county, and local government agencies have a role in the preparation, review, approval, or execution of the plan. Before finalization, a copy of the plan should be furnished to the dam agency or owner for information and comment.

(2) Evacuation plans will vary in complexity in accordance with the type and degree of occupancy in the potentially affected area. The plans may include delineation of the area to be evacuated; routes to be used; traffic control measures; shelter; methods of providing emergency transportation; special procedures for the evacuation and care of people from institutions such as hospitals, nursing homes, and prisons; procedures for securing the perimeter and for interior security of the area; procedures for the lifting of the evacuation order and reentry to the area; and details indicating which organizations are responsible for specific functions and for furnishing the materials, equipment, and personnel resources required. HEC Research Documents 19 and 20 provide example emergency plans and evacuation plans, respectively (HEC 1983a and b).

## Chapter 17 Channel Capacity Studies

### 17-1. Introduction

*a. General.* Channel capacity studies tend to focus on high flows. Flood operations for a reservoir will require operational downstream targets for nondamaging flows when excess water must be released. Nondamaging channel capacity may be defined at several locations, and the target flow may be defined at several levels. There may be lower targets for small flood events and, under extreme flood situations, the nondamaging target may cause some minor damage. Also, the nondamaging flow target may vary seasonally and depend on floodplain land use.

*b. Withstanding release rates.* Channel capacity is also concerned with the capability of the channel to withstand reservoir release rates. Of particular concern is the reach immediately downstream from the reservoir. High release rates for hydropower or flood control could damage channel banks and cause local scour and channel degradation.

*c. Channel capacity.* While flood operation may focus on maximum channel capacity, planning studies usually require stage-discharge information over the entire range of expected operations. Also, low-flow targets may be concerned with maintaining minimum downstream flow depth for navigation, recreation, or environmental goals. Channel capacity studies typically provide information on safe channel capacity and stage-discharge (rating) curves for key locations.

### 17-2. Downstream Channel Capacity

*a. Downstream channel erosion.* Water flowing over a spillway or through a sluiceway is capable of causing severe erosion of the stream bed and banks below the dam. Consequently, the dam and its appurtenant works must be so designed that harmful erosion is minimized. The outlet works for a dam usually require an energy-dissipating structure. The design may vary from an elaborate multiple-basin arrangement to a simple head wall design, depending on the number of conduits involved, the erosion resistance of the exit channel bed material, and the duration, intensity, and frequency of outlet flows. A stilling basin may be provided for outlet works when such downstream uses as navigation, irrigation, and water supply, require frequent operation or when the channel immediately downstream is easily eroded. Sections 4-2b and 4-3j of EM 1110-2-3600

provide a general discussion of energy dissipators for spillways and outlet works, respectively.

*b. Adequate capacity.* The channel downstream should have adequate capacity to carry most flows from reservoir releases. After the water has lost most of its energy in the energy-dissipating devices, it is usually transported downstream through the natural channel to its destination points. With the expected release rates, the channel should be able to resist excessive erosion and scour, and have a large enough capacity to prevent downstream flooding except during large floods.

*c. River surveys.* River surveys of various types provide the basic physical information on which river engineering planning and design are based. Survey data include information on the horizontal configuration (plan-form) of streams; characteristics of the cross sections (channel and overbank); stream slope; bed and bank materials; water discharge; sediment characteristics and discharge; water quality; and natural and cultural resources.

*d. Evaluating bank stability.* To evaluate bank stability, it is essential to understand the complex historical pattern of channel migration and bank recession of the stream and the relationship of channel changes to stream-flow. Studies of bank caving, based on survey data and aerial photographs, provide information on the progressively shifting alignment of a stream and are basic to laying out a rectified channel alignment. The concepts and evaluation procedures presented in "Stability of Flood Control Channels" (USACE 1990) are applicable to the channel capacity evaluation.

*e. Interrupted sediment flow.* A dam and reservoir project tends to interrupt the flow of sediment, which can have a significant impact on the downstream channel capacity. If the project is relatively new, the affect may not be seen by evaluating historic information or current channel conditions. The future channel capacity will depend on the long-term trends in aggradation and degradation along the river. General concepts on sediment analysis are presented in Chapter 9. *Sediment Investigations of Rivers and Reservoirs*, EM 1110-2-4000, is the primary reference for defining potential problems and analyses procedures.

*f. Downstream floodplain land use.* Channel capacity also depends on the long-term trends in downstream floodplain land use. While it is not a hydrologic problem, channel capacity studies should recognize the impact of floodplain encroachments on what is considered the nondamaging channel capacity. Anecdotal history has

shown that many Corps' projects are not able to make planned channel-capacity releases due to development and encroachments downstream.

### 17-3. Stream Rating Curve

*a. Stage-discharge relationship.* The relationship between stage and discharge, the "rating" at a gauging station, is based on field measurements with a curve fitted to plotted data of stage versus discharge. For subcritical flow, the stage-discharge relationship is controlled by the stream reach downstream of the gauge; for supercritical flow, the control is upstream of the gauge. The stage-discharge relationship is closely tied to the rate of change of discharge with time, and the rating curve for a rising stage can be different from that for the falling stage in alluvial rivers.

*b. Tailwater rating curve.* The tailwater rating curve, which gives the stage-discharge relationship of the natural stream below the dam, is dependent on the natural conditions along the stream and ordinarily cannot be altered by the spillway design or by the release characteristics. Degradation or aggradation of the river below the dam, which will affect the ultimate stage-discharge conditions, must be recognized in selecting the tailwater rating curves to be used for design. Usually, river flows which approach the maximum design discharges have never occurred, and an estimate of the tailwater rating curve must either be extrapolated from known conditions or computed on the basis of assumed or empirical criteria. Thus, the tailwater rating curve at best is only approximate, and factors of safety to compensate for variations in tailwater must be included in dependent designs.

*c. Extrapolation.* Extrapolation of rating curves is necessary when a water level is recorded below the lowest or above the highest gauged level. Where the cross section is stable, a simple method is to extend the stage-area and stage-velocity curve and, for given stage values, take the product of velocity and cross-section area to give discharge values beyond the stage values that have been gauged. Generally, water-surface profiles should be computed to develop the rating beyond the range of observed data.

*d. Rating curve shifts.* The stage-discharge relationship can vary with time, in response to degradation, aggradation, or a change in channel shape at the control section, deposition of sediment causing increased approach velocities in a weir pond, vegetation growth, or ice accumulation. Shifts in rating curves are best detected from regular gauging and become evident when several gaugings deviate from the established curve. Sediment accumulation or vegetation growth at the control will cause

deviations which increase with time, but a flood can flush away sediment and aquatic weed and cause a sudden reversal of the rating curve shift.

*e. Flow magnitude and bed material.* Stream bed configuration and roughness in alluvial channels are a function of the flow magnitude and bed material. Bed forms range from ripples and dunes in the lower regime (Froude number  $< 1.0$ ) to a smooth plane bed, to antidunes with standing waves (bed and water surface waves in phase) and with breaking waves and, finally, to a series of alternative chutes and pools in the upper regime as the Froude number increases.

*f. Upper and lower rating portions.* The large changes in resistance to flow that occur as a result of changing bed roughness affect the stage-discharge relationship. The upper portion of the rating is relatively stable if it represents the upper regime (plane-bed, transition, standing wave, or antidune regime) of bed form. The lower portion of the rating is usually in the dune regime, and the stage-discharge relationship varies almost randomly with time. Continuous definition of the stage-discharge relationship at low flow is a very difficult problem, and a mean curve for the lower regime is frequently used for gauges with shifting control.

*g. Break up of surface material.* In gravel-bed rivers, a flood may break up the armoring of the surface gravel material, leading to general degradation until a new armoring layer becomes established and ratings tend to shift between states of quasi-equilibrium. It may then be possible to shift the rating curve up or down by the change in the mean-bed level, as indicated by plots of stage and bed level versus time.

*h. Ice.* Ice at the control section may also affect the normal stage-discharge relationship. Ice effects vary with the quantity and the type of ice (surface ice, frazil ice, or anchor ice). When ice forms a jam in the channel and submerges the control or collects in sufficient amounts between the control and the gauge to increase resistance to flow, the stage-discharge relationship is affected; however, ice may form so gradually that there is little indication of its initial effects. Surface ice is the most common form and affects station ratings more frequently than frazil ice or anchor ice. The major effect of ice on a rating curve is due to backwater and may vary from day to day.

### 17-4. Water Surface Profiles

*a. Appropriate methods.* For most channel-capacity studies, water surface profiles will be computed to develop the required information. Given the technical concerns

described in the preceding section on rating curves, the selection of the appropriate method requires some evaluation of the physical system and the expected use of the information. The modeling methods are described in Chapter 8 and are presented in EM 1110-2-1416. While steady-flow water surface profiles are used in a majority of profile calculations, the unsteady flow aspects of reservoir operation or the long-term effects of changes in sediment transport may require the application of methods that capture those aspects.

*b. Further information.* The Corps, and other agencies, have accumulated considerable experience with river

systems. Appendix D, "River Modeling - Lessons Learned" (EM 1110-2-1416), provides an overview of technical issues and modeling impacts that apply to profile calculations. *Stability of Flood Control Channels* (USACE 1990) provides case examples of stream stability problems, causes, and effects. While the focus is not on reservoirs, the experience reflects the high flow conditions that are a major concern with reservoir operation. And EM 1110-2-4000 provides procedures for problem assessment and modeling. All of these documents should be reviewed prior to formulating and performing technical studies.

## Chapter 18 Real Estate and Right-of-Way Studies

### 18-1. Introduction

*a. General.* This chapter provides guidance on the application of hydrologic engineering principles to determine real estate acquisition requirements for reservoir projects. Topics include selection of the analysis method, potential problems, evaluation criteria, and references associated with the acquisition of real estate for reservoir projects developed by the Corps of Engineers.

*b. Related documents.* Real estate reporting requirements associated with feasibility reports, General Design Memoranda, and Real Estate Design Memoranda are set forth in ER 405-1-12. Real estate reporting requirements associated with the acquisition of lands downstream from spillways are set forth in ER 1110-2-1451, paragraph 9.

### 18-2. Definition of Terms

A list of terms and definitions used in this chapter is as follows:

*a. Project design sediment.* The volume and distribution of sediment deposited in a reservoir over the life of the project.

*b. Land acquisition flood.* A hypothetical or recorded flood event used to determine requirements for real estate acquisition.

*c. Full pool.* The maximum reservoir elevation for storing water for allocated project purposes.

*d. Induced surcharge.* Storage created in a reservoir above the top of flood control pool by regulating outflows during flood events.

*e. Envelope curve.* A curve which connects the high points of intersection of preproject and postproject water-surface profiles.

*f. Guide taking line.* A contour line used as a guide for land acquisition in the reservoir area. (Also referred to as the guide contour line or guide acquisition line.)

### 18-3. Real Estate Acquisition Policies for Reservoirs

*a. Basic policies.* Basic policies and procedures related to the acquisition of lands for reservoirs are presented in ER 405-1-12. Paragraph 2-12 of ER 405-1-12 states that, "Under the Joint Policy the Corps will take an adequate interest in lands, including areas required for public access, to accomplish all the authorized purposes of the project and thereby obtain maximum public benefits therefrom." Paragraph 2-12.a(2) further states that land to be acquired in fee shall include, "lands below a guide contour line...established with a reasonable freeboard allowance above the top pool elevation for storing water for flood control, navigation, power, irrigation, and other purposes, referred to in this paragraph as "full pool" elevation. In nonurban areas generally, this freeboard allowance will be established to include allowances for induced surcharge operations plus a reasonable additional freeboard to provide for adverse effects of saturation, wave action and bank erosion."

*b. Considering factors.* Factors such as estimated frequency of occurrence, probable accuracy of estimates, and relocation costs will be taken into consideration. Where freeboard does not provide a minimum of 300 ft horizontally from the conservation pool, defined as the top of all planned storage not devoted exclusively to flood-control storage, then the guide acquisition line will be increased to that extent. In the vicinity of urban communities or other areas of highly concentrated developments, the total freeboard allowance between the full pool elevation and the acquisition line may be greater than prescribed for nonurban areas generally. Also, there should be sufficient distance to assure that major hazards to life or unusually severe property damages would not result from floods up to the magnitude of the SPF. In such circumstances, however, consideration may be given to easements rather than fee acquisition for select sections if found to be in the public interest. However, when the project design provides a high level spillway, the crest of which for economy of construction is considerably higher than the storage elevation required to regulate the reservoir design flood, the upper level of fee acquisition will normally be at least equal to the top elevation of spillway gates or crest elevation of ungated spillway, and may exceed this elevation if necessary to conform with other criteria prescribed.

#### 18-4. Hydrologic Evaluations

*a. Development of land acquisition flood.* To establish a reasonable surcharge allowance above the top pool elevation, a land acquisition flood, which includes the effects of any upstream reservoirs, should be selected and routed through the project to determine the impact on the establishment of the guide acquisition line.

*b. Nonurban areas.* In nonurban areas, the land acquisition flood should be selected from an evaluation of a range of floods with various frequencies of occurrence. The impact of induced surcharge operations on existing and future developments, hazards to life, land use, and relocations must be evaluated. The land acquisition flood will be chosen based on an evaluation of the risk and uncertainty associated with each of these frequency events. Basic considerations to be addressed during the land acquisition flood selection process should include the credibility of the analysis, identification and significance of risk, costs and benefits, and legal, social, and political ramifications.

*c. Urban areas.* In urban areas or other areas with highly concentrated areas of development, the SPF will be used for the land acquisition flood.

*d. Project design sedimentation volume.* Project capacity data should be adjusted for projected sediment volumes when routing the land acquisition flood. Project design sediment should be based on appropriate rates of sedimentation for the project area for the life of the project.

#### 18-5. Water Surface Profile Computations

*a. Backwater model development.* A basic backwater model should be developed for the project area from the proposed flat pool area through the headwater area where impacts of the proposed reservoir are expected to be significant. The model should reflect appropriate cross-sectional data and include parameters based on historical flood discharges and high water marks. EM 1110-2-1416 presents the model requirements and calibration procedures.

*b. Preproject profiles.* A series of preproject water surface profiles should be developed utilizing preproject cross-section geometry, calibrated Manning's "n" values, and appropriate starting water surface elevations for the initial cross section. Flow rates used in the water surface profile computations should be selected from the peak and recession side of the land acquisition flood hydrograph.

*c. Postproject profiles.* A series of water surface profiles shall be developed utilizing the postproject cross sections which are adjusted to reflect project design sedimentation over the life of the project. Manning's roughness coefficients are based on adjusted preproject roughness coefficients to account for factors such as vegetation and land use changes which decrease hydraulic conveyance. Agricultural lands existing in the headwater areas prior to land purchases will likely revert to forested areas some years after the reservoir is filled. Preproject flow rates and coincident reservoir pool elevations from land acquisition flood routing should be used to compute postproject profiles.

*d. Project design sedimentation distribution.* Post-project cross-section geometry must be adjusted to reflect the impacts of sedimentation over the life of the project. Sedimentation problems associated with reservoir projects and methods of analysis to address sediment volumes and distributions are given in Chapter 5 of EM 1110-2-4000.

#### 18-6. Development of an Envelope Curve

The development of an envelope curve is based on pre-project and postproject water-surface profiles. A selected discharge from the land acquisition flood is used to compute a preproject and a postproject profile. A point of intersection is established where the profiles are within 1 ft of each other. The point of intersection is placed at the elevation of the higher of the two profiles. A series of points of intersection are derived from water-surface profile computations utilizing a range of selected discharges from the land acquisition flood. A curve is drawn through the series of points of intersection to establish the envelope curve.

#### 18-7. Evaluations to Determine Guide Taking Lines (GTL)

*a. Land acquisition flat pool.* The land acquisition flat pool of a reservoir project is established by the maximum pool elevation designated for storing water for allocated project purposes to include induced surcharge storage and is not impacted by the backwater effects of main stream or tributary inflows. In flat pool areas, the elevations of the GTL are based on the flat pool elevation and a freeboard allowance to account for adverse effects of saturation, bank erosion, and wave action.

*b. Headwater areas.* In headwater areas, the GTL may be based on the envelope curve elevations and appropriate allowances to prevent damages associated with saturation, bank erosion, and wave action.

c. *Flood-control projects.* The selection of an appropriate land acquisition flood for flood-control projects located in rural areas should be based on an elevation of a range of frequency flood events. The land acquisition flood selection for flood-control projects in rural locations must include regulation by upstream reservoirs and reflect postproject conditions which minimize adverse impacts within the project area resulting from induced flood elevations and duration of flooding. In highly developed areas along the perimeter of flood-control projects, the SPF should be used for land acquisition. An envelope curve can be developed from the land acquisition flood routings and water-surface profile computations for preproject and postproject conditions. The land acquisition GTL may be established from the envelope curve and appropriate allowances for reservoir disturbances.

d. *Nonflood-control projects.* Nonflood control projects may be any combination of purposes such as water supply, hydropower, recreation, navigation and irrigation. The land acquisition flood selection process for nonflood-control projects located in rural areas is based on an evaluation of a range of frequency floods and is used to determine postproject flood elevations and duration of flooding in the project area. As with flood-control projects, regulation of flows by upstream reservoirs must be incorporated in the development process. The land acquisition flood used to evaluate real estate acquisitions in rural areas should reflect postproject conditions which minimize adverse impacts. The land acquisition flood for developed areas should be the SPF. The maximum pool elevation designed for storing water for allocated project purposes is used in the development of the land acquisition flood routing. An envelope curve based on preproject and postproject water-surface profiles utilizing project design sedimentation and distribution should be developed. The envelope curve and appropriate allowances for reservoir disturbances may be used to establish the land acquisition GTL.

## 18-8. Acquisitions of Lands for Reservoir Projects

Land acquisition policies of the Department of the Army governing acquisition of land for reservoir projects is published in ER 405-1-12, Change 6, dated 2 January 1979. Paragraph is as follows:

*Joint Land Acquisition Policy for Reservoir Projects.*  
*The joint policies of the Department of the Interior and Department of the Army, governing the acquisition of land for reservoir projects, are published in the Federal*

*Register, dated 22 February 1962, Volume 27, page 1734. On July 1966, the Joint Policy was again published in 31, F.R. 9108, as follows:*

### *JOINT POLICIES OF THE DEPARTMENTS OF THE INTERIOR AND OF THE ARMY RELATIVE TO RESERVOIR PROJECT LANDS*

*"A joint policy statement of the Department of the Interior and the Department of the Army was inadvertently issued as a notice in 27 F.R. 1734. Publication should have been made as a final rule replacing regulations then appearing in 43 CFR part 8. The policy as it appears in 27 F.R. 1734 has been the policy of the Department of the Interior and the Department of the Army since its publication as a Notice and is now codified as set forth below.*

#### Section

- 8.0 *Acquisition of lands for reservoir projects*
- 8.1 *Lands for reservoir construction and operation*
- 8.2 *Additional lands for correlative purposes*
- 8.3 *Easements*
- 8.4 *Blocking out*
- 8.5 *Mineral rights*
- 8.6 *Building*

*Authority: The provisions of this Part 8 issued under Sec. 7, 32 Stat., 389, Sec. 14, 53 Stat. 1197, 43 U.S.C. 421, 389.*

*8.0 Acquisition of Lands for Reservoir Projects.*  
*Insofar as permitted by law, it is the policy of the Departments of the Interior and of the Army to acquire, as a part of reservoir project construction, adequate interest in lands necessary for the realization of optimum values for all purposes including additional land areas to assure full realization of optimum present and future outdoor recreational and fish and wildlife potentials of each reservoir.*

*8.1 Lands for Reservoir Construction and Operation.*  
*The fee title will be acquired to the following:*

a) *Lands necessary for permanent structures.*

b) *Lands below the maximum flowage line of the reservoir including lands below a selected freeboard where necessary to safeguard against the effects of saturation, wave action, and bank erosion and to permit induced surcharge operation.*

c) *Lands needed to provide for public access to the maximum flowage line, as described in Paragraph 1b, or for operation and maintenance of the project.*

8.2 Additional Lands for Correlative Purposes. *The fee title will be acquired for the following:*

a) *Such lands as are needed to meet present and future requirements for fish and wildlife as are determined pursuant to the Fish and Wildlife Coordination Act.*

b) *Such lands as are needed to meet present and future public requirements for outdoor recreation, as may be authorized by Congress.*

8.3 Easements. *Easements in lieu of fee title may be taken only for lands that meet all of the following conditions:*

a) *Lands lying above the storage pool,*

b) *Lands in remote portions of the project area,*

c) *Lands determined to be of no substantial value for protection or enhancement of fish and wildlife resources, or for public outdoor recreation,*

d) *It is to the financial advantage of the Government to take easements in lieu of fee title.*

8.4 Blocking Out. *Blocking out will be accomplished in accordance with sound real estate practices, for example, on minor sectional subdivision lines; and normally, land will not be acquired to avoid severance damage if the owner will waive such damage.*

8.5 Mineral Rights. *Mineral, oil and gas rights will not be acquired except where the development thereof would interfere with project purposes, but mineral rights not acquired will be subordinated to the Government's right to regulate their development in a manner that will not interfere with the primary purposes of the project, including public access.*

8.6 Buildings. *Buildings for human occupancy as well as other structures which would interfere with the operation of the project for any project purpose will be prohibited on reservoir project lands."*

## **18-9. Acquisition of Lands Downstream from Spillways for Hydrologic Safety Purposes**

a. *General.* A real estate interest will be acquired downstream of dam and lake projects to assure adequate security for the general public in areas downstream from spillways. Real estate interests must be obtained for downstream areas where spillway discharges create or significantly increase a hazardous condition.

b. *Evaluation criteria.* Combinations of flood events and flood conditions which result in a hazardous condition or increase the hazard from the preproject to postproject flood conditions are determined for areas downstream from the spillway. These combinations of flood events and flood conditions are identified as critical conditions.

c. *Flood events and conditions.* Flood events up to the magnitude of the spillway design flood are evaluated for preproject and postproject conditions for areas downstream from the spillway. Flood conditions to be analyzed include flooded area, depth of flooding, duration, velocities, debris, and erosion.

d. *Hazardous and nonhazardous conditions.* The imposed critical conditions are analyzed to determine if these conditions are hazardous or nonhazardous. Non-hazardous areas are characterized by the following criteria:

(1) Flood depths do not exceed 2 ft in urban and rural areas.

(2) Flood depths are essentially nondamaging to urban property.

(3) Flood durations do not exceed 3 hr in urban areas and 24 hr in agricultural areas.

(4) Velocities do not exceed 4 fps.

(5) Debris and erosion potential are minimal.

(6) Imposed flood conditions would be infrequent. The exceedance frequency should be less than 1 percent.